Analysis of Behaviour of U-Girder Bridge Decks

V Raju[1], Devdas Menon[2]
[1] Research Scholar, Indian Institute of Technology Madras, Chennai, India
Email: vrgd.raju@gmail.com
[2] Professor, Indian Institute of Technology Madras, Chennai, India
Email: dmenon@iitm.ac.in

***Abstract*—The concept of U-shaped bridge girder is now being increasingly adopted in urban metro rail projects and for replacing old bridges where there is a constraint on vertical clearance. These bridge decks are commonly designed in practice using simplified methods that assume beam action of the webs in the longitudinal direction and similar flexural action of the deck slab in the transverse direction. However, such assumptions can lead to errors. This paper attempts to assess the extent of error in the simplified analysis, by comparing the results with a more rigorous three-dimensional finite element analysis (3DFEA). A typical prototype railway bridge girder has been taken as a case study. The results of the 3DFEA, in terms of load-deflection plots, have been validated by field testing.**

***Index Terms*— U-girder bridge deck, Simplified methods, Three-dimensional finite element analysis**

I. Introduction

The U-shaped girder bridge (also called 'channel bridge') is a relatively new and innovative concept in bridge deck design. U-shaped girder is appropriate when a new or modified alignment structure requires an increase in the vertical clearance beneath the bridge. The bridge deck, made of prestressed concrete (PSC), has other important advantages, such as protection against traffic noise pollution, aesthetic appearance, reduced construction time, durability and economy. This concept can be used for overpasses, under-crossings, viaducts, etc.

A. Description of U-girder bridge concept

Structurally, the U-shaped girder bridge can be viewed as the conventional 'single-cell box girder' with its top flange removed, as shown in Fig. 1. The two webs are configured as beams positioned above and on either side of the deck surface. The webs and the deck slab are post-tensioned with longitudinal tendons anchored at the two ends of the bridge deck (with suitable 'end blocks'). The longitudinal stiffness and strength are obtained from the two webs as well as the connecting passageway slab spans between the webs. The resulting requirement for the depth of girder section below the passageway level is very less than that required for convectional beam-and slab type designs, as shown in Fig. 2, and herein lies its main functional advantage. The U-girder is essentially a 'through' type girder where the train passage occurs on the soffit slab; the side cantilevers serve as 'keyman' pathways (for maintenance).

[2]Devdas Menon (corresponding author), Professor, Structural Engineering Division, Indian Institute of Technology Madras, Chennai, Inida.

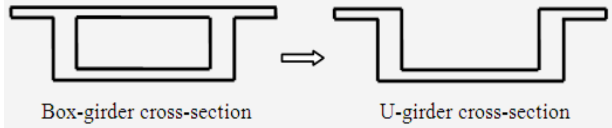


Figure 1. U-shaped girder versus Box-girder

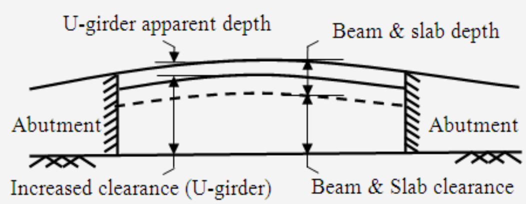


Figure 2. Comparison of U-girder concept with conventional "beam and slab" construction

B. Evolution of 'Channel Bridge'

The precast segmental concrete "channel" or "U-shaped" bridge was first developed by Jean Muller in 1990s for the Champfeuillaet Overpass Bridge in France. Subsequently, in the mid-1990s, there was an extensive research evaluation programme carried out in USA by the Highway Innovative Technology Evaluation Centre (HITEC). Between 2001 and 2003, the Sorell Causeway Viaduct was built in Australia. It became the first channel bridge viaduct built in the world [1].

The specialist rail consultancy firm, Systra, has developed a precast prestressed concrete U-shaped type bridge based on the original channel bridge constructed in France. The Wodonga Rail Bypass project in Austria, designed by Systra uses a simplified U-shaped bridge concept as shown in Fig. 3; an international patent has been taken for this concept [2].

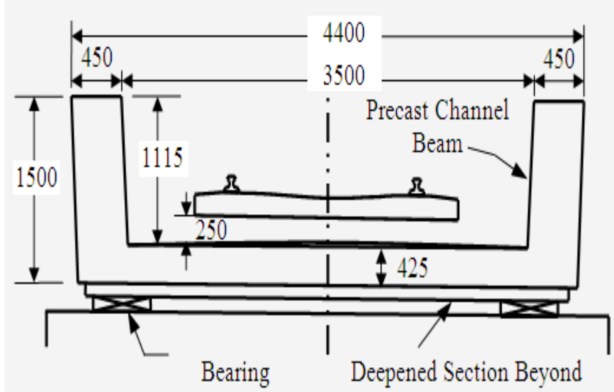


Figure 3. Cross section for the Wodonga Rail Bypass in Australia

DOI: 01.IJTUD.01.01.32

II. Need for present study

The U-shaped girders are analysed and designed in practice using simplified methods. In simplified methods of analysis, the bridge deck behaviour is conveniently divided into longitudinal analysis and transverse analysis. In longitudinal analysis, the whole U-girder is treated as a simply supported beam in the longitudinal direction; it is known as 'simple beam analysis (SBA)'. For bending in the transverse direction, the deck slab alone is analysed as being simply supported between the two webs, as shown in Fig. 4. Such analyses do not account for the interaction between the longitudinal and transverse bending, as well as warping, distortion and shear lag effects under possible eccentric loading (encountered in two-lane railway decks and in highway decks). In the present paper, the study is limited to single track railway deck, now being used in railway bridges in India. The results obtained by simplified methods are compared with the more rigorous three-dimensional finite element method of analysis (3DFEA).

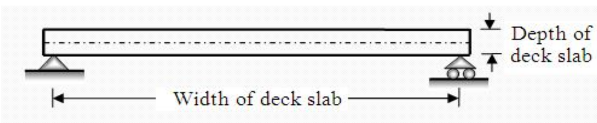


Figure 4. Deck slab is modelled as a beam for transverse analysis

III. Details of model bridge deck

A prototype U-shaped railway bridge deck is considered for analysis, which is a real structure at Villupuram, Tamil Nadu, designed and constructed recently. The effective span and overall span of the bridge are 18.5 m and 19.7 m respectively between the piers. The overall width of the cross section is 6.7 m and the overall depth is 1.8 m. The section of the bridge deck is a U-girder, which comprises two webs (0.6 m wide and 1.3 m deep) with a deck slab (0.5 m thick) having a clear spacing of 4.5 m between the webs, to cater to a single lane Modified Broad Gauge (MBG). The projections on top of the webs have an overall width of 1.1 m, acts as footpaths or keyman walkways (Fig. 5). The proposed design is that of a post-tensioned girder with 15 parabolic profile cables, each cable comprising 12 strands (each strand having an area of 98.7 mm^2). The arrangement of cables at the mid-span location is shown in Fig. 5. At the pier locations, the U-girder bridge deck is supported on elastomeric bearings, under each web.

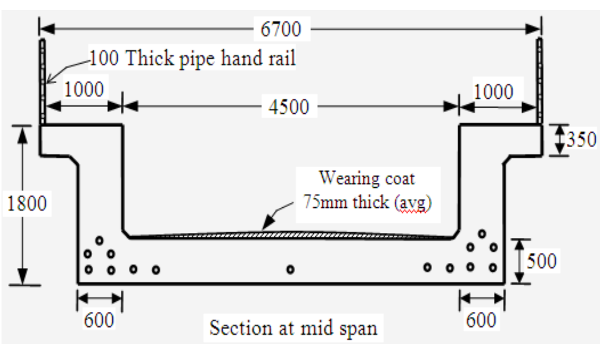


Figure 5. Cross section of U-girder bridge deck with cable locations

IV. Design basis

The loads considered for analysis and design of this bridge are based on Indian Railway Standards (IRS) Bridge Rules [3]. The analysis had been carried out by the designers using the concepts of simplified analysis (described earlier) for the longitudinal and transverse actions. The prestressing in the bridge is designed for 'no tension' conditions under service loads according to IRS Concrete Bridge Code [4]. In the present study, a rigorous analysis of the bridge has been carried out using three dimensional finite element analysis (3DFEA) under service loads.

A. Finite element model descriptiont

The linear finite element model is developed using SAP 2000 package [5], a commonly available finite element program. All the components of girder are modelled using four-noded quadrilateral shell element and with aspect ratio of around one. To ensure accuracy of results, a convergence study of the solution has been performed. Material properties are specified as isotropic and the following values are used in modelling: Modulus of elasticity (M45 Grade), E = 3.35×10^7 MPa, Poisson's ratio = 0.2 and Unit weight = 24kN/m^3. The prestressing cables are modelled as tendon loads with tendon section element using feature in SAP 2000. A 3D view of one-half of the bridge model is shown in Fig. 6. The supports are modelled as linear elastic translational springs with specified elastomeric bearing stiffness. Live (EUDL) loads are assigned as uniformly surface pressure on top of bridge deck. Prestressing losses are manually calculated and the corresponding effective prestressing force is applied as loads at the ends of the parabolic tendon, using the 'load balancing' concept.

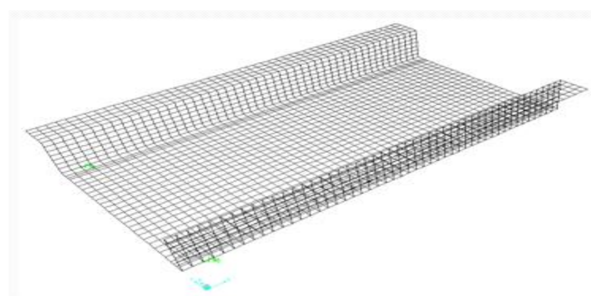


Figure 6. Half span of a 3D model of U-girder in SAP2000

V. Field testing

It is assumed that the 3DFEA method is more rigorous and accurate, compared to the simplified methods of analysis, but the accuracy of the results need to be verified using available experimental results. This has been facilitated in this instance by the field testing carried out on the prototype U-girder bridge deck.

Load testing was conducted at the Villupuram site of the bridge. The test carried out mainly to assess the flexural capacity of the structure at working loads in the elastic range, through measuring the deflections of the super structure. The static load test was performed using sand filled bags.

DOI: 01.IJTUD.01.01.32

The load testing was conducted as per IRS Concrete Bridge Code [4].

A. Test method

A test load is calculated as per IRS code, clause 18.2.3 [4], 4808 kN, for limit states of deflection. Dial gauge are located with independent staging at three different locations along span length, i.e., mid-span and under each support, as shown in Fig. 7. The deflections of the girder caused due to variations in ambient temperature are monitored at one hour intervals for 24 hour. Deflections are measured by using mechanical dial gauges together with the ambient temperature at each stage of loading and unloading respectively. A view of the bridge span loaded with sand bags is shown in Fig. 8. The total load was applied in four increments of 1202 kN each, and deflections measured at these four stages of loading as well as unloading. All deflection data was corrected for temperature effects.

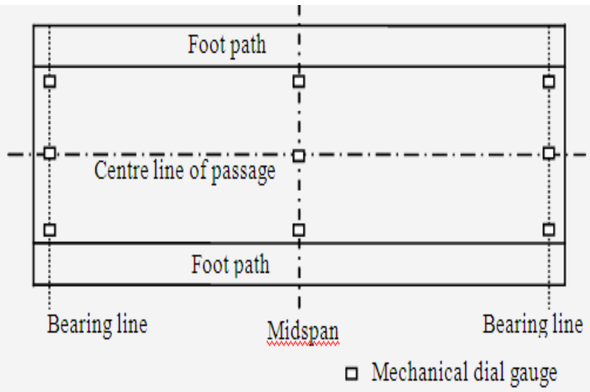


Figure 7. Dial gauge locations along span and across width of girder in plan view

Figure 8. Load applied with sand bags

Fig. 9 shows the load-deflection curves at the mid-span section of the U-girder, the deflection being measured at the centre of deck slab and under the webs. To facilitate comparison with the results of 3DFEA, the effect of pre-camber in the deck on account of prestressing, is also included in Fig. 9, after compensating dead load deflections. The results show close correspondence between the experimental and numerical results. The slight non-linearity in the plots is attributable to the non-linear stiffness in the elastomeric bearings.

VI. Behaviour of U-girder

It is instructive to compare the behaviour of the U-girder, as obtained from simplified methods, with the more accurate 3DFEA results for the bridge under consideration. The dashed line in Fig. 10(*a*) shows the deflected shape (as per simple beam analysis) of the U-girder section. It is seen that in the simplified longitudinal analysis, the deflection is implicitly assumed to be constant throughout the cross section. In Fig. 10(*b*), the dashed line indicates the actual deflected shape as captured by the 3DFEA, which includes transverse bending of plates along with combined effect of longitudinal and transverse bending curvatures.

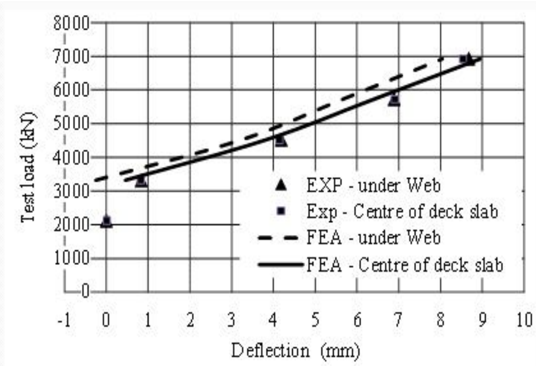
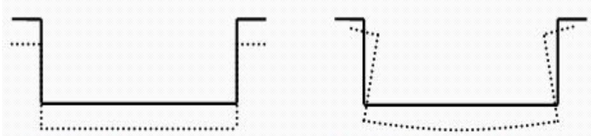


Figure 9. Load-deflection curves across the bottom deck



(a) Simple beam analysis (b) 3DFEA (actual behaviour)

Figure 10. Deflection of cross-section

Fig. 11 shows comparison of the normal force distribution as per simple beam analysis and 3DFEA. It is seen that the 3DFEA brings out the nonlinear variation of longitudinal forces across the width of the section in the flanges (deck slab and cantilevers), which is not captured by simple beam analysis. This nonlinear variation induces slightly higher stresses at the web-flange junction, compared to the middle of the flange, and is attributable to shear lag effect in flanges.

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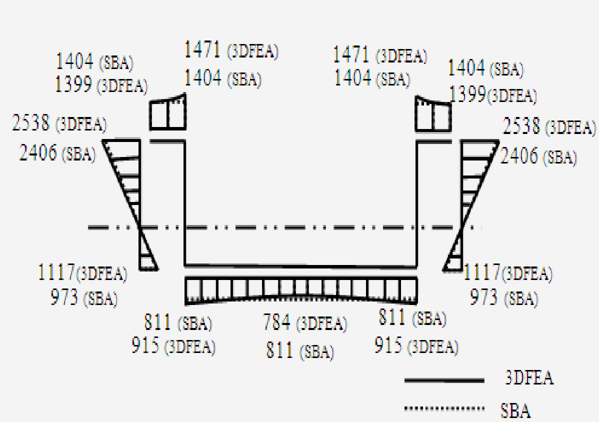

Figure 11. Comparison of normal force (kN/m) distribution by 3DFEA versus SBA

The percentage of error, based on 3DFEA, is calculated from the following equation:

$$\text{Percentage of error} = \frac{(\text{Simplified analysis results} - \text{3DFEA results})}{(\text{3DFEA results})} \times 100 \quad (1)$$

As per Eq. (1), the negative sign indicates that the simplified method of analysis is under-estimating while the positive sign indicates that the simplified method of analysis is over-estimating, as comparison with 3DFEA.

Fig. 12 shows percentage of error in longitudinal force estimated by simple beam analysis over 3DFEA, at mid-span. From this figure (Fig. 11), it can be found that simple beam analysis under-estimate the maximum stress in the web-deck slab junction by about 12 percent; the disparity elsewhere is within 5 percent.

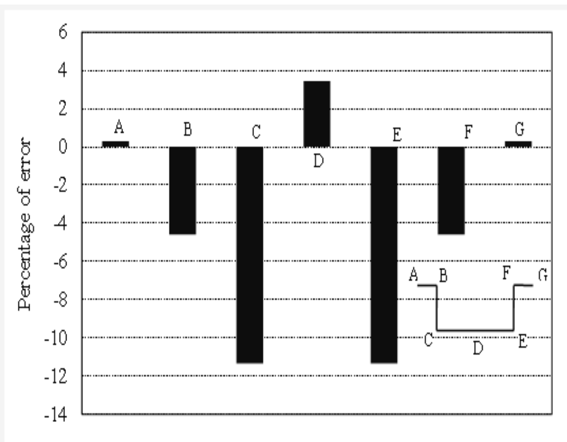

Figure 12. Percentage of error in longitudinal forces estimated by simple beam analysis at mid span

Fig. 13 shows the comparison of the transverse bending moments as predicted by transverse analysis and 3DFEA at the mid-span of U-girder. The transverse moments obtained by 3DFEA and transverse analysis are shown (in Fig. 13) with solid line and dotted line respectively. The sagging moments in the slab are over-estimated by about 9 percent in the simplified analysis. According to 3DFEA, some marginal hogging moments also get induced near the web-slab junctions and in the webs; but these are completely missed out in the simplified method of analysis. Fig. 14 shows the variation of transverse bending moment along the span at centre of deck slab and web-deck slab junction. Transverse bending moments obtained by 3DFEA in the web and cantilever portion are very less (Fig. 13). Fortunately, the nominal reinforcement provided in the transverse direction in the webs and top of slab is adequate to give the necessary flexural strength due to this transverse bending.

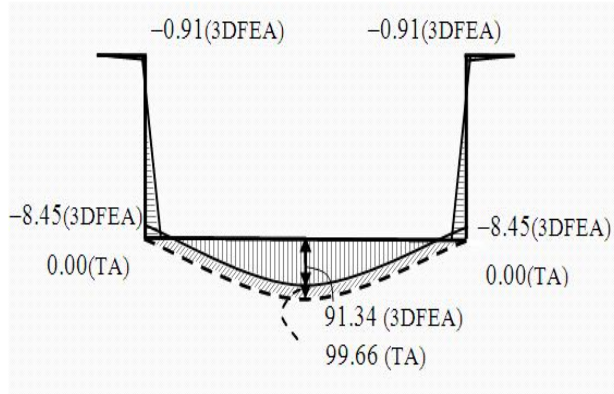

Figure 13. Comparison of transverse bending moements (kN-m/m) by 3DFEA versus transverse analysis

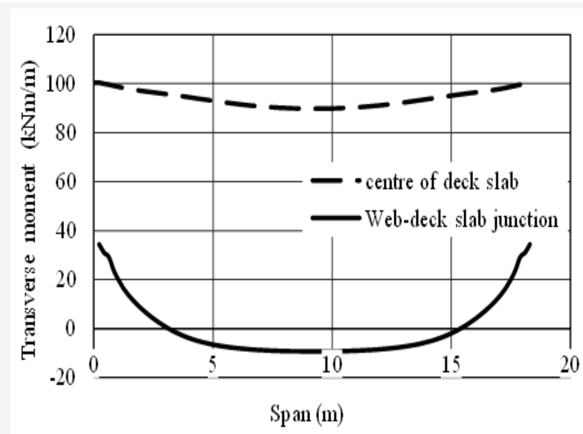

Figure 14. Variation of transverse moments along span

Conclusions

The longitudinal and transverse behaviour of a simply supported U-girder bridge deck have been studied using the simplified methods of analysis (used in practice), and compared with more accurate 3DFEA results. The deflections in the bridge girder, as predicted by 3DFEA, under various stages of loading, has been validated against field test results on a typical prototype U-girder railway bridge, recently constructed. It is seen that simple beam analysis generally predicts good results, except for some local stress concentrations. The simplified longitudinal analysis under-estimates the maximum stress in the web-deck slab junction by about 12 percent, because it is not able to capture the effects arising from shear lag and transverse bending. The

simplified transverse analysis over-estimates the sagging moments in the deck slab by about 9 percent, but fails to capture the hogging moments near the webs and the transverse bending in the webs. These errors can be compensated for by the designer, by adopting appropriate corrections while designing and detailing.

However, the use of 3DFEA is recommended in cases where eccentric loading occurs on the U-girder, as in double track railway and highway bridges, because the effects of torsion and distortion (not accounted for in simplified analysis) can be significant.

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DOI: 01.IJTUD.01.01.32